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INSTITUTE OF ENGINEERING RESEARCH
BERKELEY, CALIFORNIA

SERIES 3
ISSUE 361

WAVE RESEARCH LABORATORY

FLOW OVER REEFS AND STRUCTURES BY WAVE ACTION

BY

OSVALD SIBUL

JANUARY 1954

UNIVERSITY OF CALIFORNIA

University of California
College of Engineering
Submitted under Contract N7onr-295(28)
with the Office of Naval Research

Institute of Engineering Research
Waves Research Laboratory
Technical Report
Series 3 Issue 361

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Abstract

The discharge of water caused by wave action on reefs and structures with impermeable uniformly sloping sides was measured for various elevations of crest with respect to the still-water level. The experiments were completed for side-slopes of 1:2 and 1:3 and for smooth and rough surfaces of the barrier. The wave steepness, H/L, and the relative depth, d/L, were used as parameters and dimensionless plots were made for the discharge, (QT/H^2), as a function of relative elevation, h/H, of the crest of the barrier.

Introduction

The crests of natural reefs generally are at or below the mean tide level. It has long been observed that waves breaking over a reef will "pump" water over the reef -- this water then usually flows along the lee side of the reef and returns to the open sea through a gap or inlet. In times of "high" wave action the currents through such inlets may be relatively large and even make navigation of surface craft quite hazardous.

In the case of man-made structures such as levees and dikes, the crests are located well above the maximum expected high water. Extreme tides, wind tides, or flood conditions, however, may raise the water stage to such a level that wave action on the windward side might cause wave run-up to be excessive and pump an appreciable quantity of water over the structure and thereby create a flood condition in the area protected by the structure. A disaster of this type occurred in Holland and Great Britain in February 1953 where the loss in property and human lives was enormous. Such structures must be designed to withstand not only the static, dynamic and erosive forces, but the crest elevations also must be sufficiently high to prevent overflow of water by wave action.

There appears to be little data available to permit a prediction of wave run-up on shore protection structures under a wide variety of conditions; the available information indicates, however, that wave run-up might amount to twice the wave height or sometimes even more (Granthem, 1953). Considering high tide ranges for some areas, and considerable wind tides (Keulegan, 1951; and Corps of Engineers, 1951), it is possible that very high structures may be necessary for some localities in order to prevent overtopping. Such high structures usually are very expensive and a reduction of design height by only a foot might result in considerable savings in construction costs.

As in the case of wave run-up, there is little information available on the quantity of water that might be pumped over various types of structures by wave action. The first experiments in this direction were completed by

* Presented at the Pacific Southwest Meeting, American Geophysical Union, 29 and 30 January 1954.

Saville and Caldwell (1953) for a vertical faced seawall. The results probably will not apply for other types of structures. To fill this gap in our knowledge, the series of tests described below were performed in the Wave Research Laboratory of the University of California. The tests were conducted on structures with a uniformly sloping impermeable face. Two different side-slopes were investigated (namely 1:2 and 1:3) under a variety of wave conditions. The results are given as dimensionless plots of the variables involved.

Laboratory Equipment and Procedure

The experiments were conducted in a glass-walled wave channel 1 foot wide by 3 feet deep by 60 feet long. Waves were generated by a flapper type generator mounted at one end of the channel. Both the amplitude and the period of the flapper movement were adjustable. The period could be varied between approximately 0.4 seconds and 2 seconds, and the maximum wave height was about 0.5 foot. The water depth during all experiments was 1.69 feet.

The model was located approximately 45 feet from the wave generator and was built in the form of a sheet-iron box as shown in Fig. 1. The front end of the model, representing the structure, consisted of a sloping board. The slope of the board could be varied by rotating it around point A (Figure 1), and the crest elevation of the structure could be placed at various positions above or below the still-water level. Special care was taken to seal all the cracks between the structure and the channel walls, so that water circulation could not occur. The roughness of the structure was varied by using different plates fastened on the surface of the sloping board. The smooth surface was represented by a smooth aluminum sheet, while for a rough surface transverse wood strips were fastened to the aluminum sheet (see Figure 2). The relative roughness was found to be equal to 0.036 foot for the smooth surface and 0.13 foot for the rough surface (Johnson, 1944).

Water which was pumped over the structure by wave action was caught in a measuring tank, and the discharge was determined gravimetrically. The tank was arranged so that the water passing over the structure could be diverted either into the measuring tank or into a waste area. The arrangement was necessary to permit the measurement of discharge for any chosen wave group or for a fixed length of time. The handle of the diversion flap was arranged so that a contact to an electrically operated clock was closed the moment the flap was opened to the measuring tank and the contact out again when the flap was closed. This arrangement enabled a very accurate time measurement. The clock was graduated to give a least reading of 1/100 of a second. The water in the waste tank was returned to the channel at the backside of the model by means of an auxiliary pump which operated continuously. This water could return again to the main channel and thereby maintain the depth of water therein a constant. A wave absorber was placed between the structure and the measuring tank so that the flow over edge C (Fig. 1) would be more regular and not affected by the turbulence of the breaking waves. The absorber also was particularly important for the case where the edge B of the structure was below the still water level.

The wave heights and periods were measured about two to four wave lengths seaward of the structure. Wave heights were measured by a double wire resistance element and a Brush recorder.

Procedure: The structure was given a desired slope, θ, and the edge, B, set to a certain height, h, above (or below) still-water level. The distance, h, was taken as positive when the crest of the structure was above the still-water level, and as negative when below the still-water level. The average value for h was obtained by measuring the elevation with a point gage at at least three locations across the channel. In order to permit equilibrium conditions to exist after starting the wave generator, the water pumped over the structure by the first three or four waves was diverted to the waste area and then the measurement of water pumped was made of the following 3 to 10 waves according to the capacity of the measuring tank. Special care was taken to limit the measurement to the time before reflected waves from the structure could reach the generator and return to the structure. After the wave generator was stopped, the volume of the pumped water, the number of waves acting on the structure and the duration of the measurement as obtained from the electrically operated clock were recorded. The measurements were repeated five times for each condition. In general there was very little difference in measurements between the runs made under the same conditions. The wave heights and periods were obtained from Brush records, disregarding the first three or four waves and those which occurred after reflection effects became noticeable. By examining the wave records it was relatively easy to recognize the point where the reflection from the structure started to change the wave characteristics. The wave heights and periods as used represent the wave condition in the given depth of water as though no structure was present. A set of experiments usually was started with the crest of the structure approximately one wave height below the still-water level, and the crest was raised for the following runs in increments of approximately 0.05 ft. until a height, h, was reached where no appreciable overtopping of the structure occurred. Usually 40 to 50 single runs were completed for each set of experiments. Before starting each succeeding run the water surface was permitted to quiet completely.

The general test procedure was to select a given wave condition (H and T held constant) and then measure the discharge of water pumped over the structure for various elevations, h, of the crest. Tests were conducted under the following wave conditions:

1. For a constant wave steepness H/L of about 0.055, tests were made with d/L values of 0.339 and 0.589. The discharge passing over the structure was determined for several relative crest elevations, h/H.

2. For a constant relative depth d/L of 0.236, tests were made with wave steepnesses H/L of 0.0108, and 0.0182, and 0.0293. Here again the discharge passing over the structure was determined for several relative crest elevations, h/H.

For each of the above wave conditions tests were made with face slopes of 1:2 and 1:3 and with both smooth and rough surfaces. A summary of test conditions is given in Table 1. The experimental data from the various runs are plotted in Figures 3 and 4.

TABLE 1

Run No.	Slope of Structure	Surface of Structure	Depth of Water d, ft	Wave Period T sec.	Wave Length L ft.	Height H ft.	H/L	d/L	Maximum Q* ft^3/sec/ft.	Max. "overtopping per crest of wave per ft. of structure Q_w	Maximum $\frac{QT}{H^2}$ *	Maximum Uprush $\frac{h}{H}$
35-84	1:3	Smooth	1.69	0.75	2.87	0.16	0.0558	0.589	0.0200	0.0153	0.60	1.05
35-84	1:3	Rough	1.69	"	"	"	"	"	0.0160	0.0120	0.47	0.85
85-131	1:3	Rough	1.69	1.00	4.98	0.26	0.0522	0.340	0.0500	0.0500	0.74	1.10
132-173	1:3	Smooth	1.69	"	"	"	"	"	0.0570	0.0570	0.84	1.60
174-213	1:3	Smooth	1.69	1.25	7.17	0.130	0.0181	0.236	0.0276	0.0346	2.05	2.40
214-242	1:3	Rough	1.69	"	"	"	"	"	0.0227	0.0284	1.68	1.90
243-281	1:3	Rough	1.69	"	"	0.21	0.0293	"	0.0550	0.0686	1.56	1.55
282-321	1:3	Smooth	1.69	"	"	"	"	"	0.0585	0.0732	1.66	1.90
322-349	1:3	Smooth	1.69	"	"	0.0775	0.0108	"	0.0113	0.0131	2.35	1.85
350-377	1:3	Rough	1.69	"	"	"	"	"	0.0096	0.0120	2.00	1.50
378-399	1:2	Smooth	1.69	1.25	7.17	0.0775	0.0108	0.236	0.0092	0.0115	1.92	1.40
400-427	1:2	Rough	1.69	"	"	"	"	"	0.0068	0.0085	1.42	1.10
428-452	1:2	Rough	1.69	"	"	0.130	0.0182	"	0.0208	0.0260	1.54	1.50
453-474	1:2	Smooth	1.69	"	"	"	"	"	0.0235	0.0294	1.74	1.80
475-500	1:2	Smooth	1.69	"	"	0.210	0.0293	"	0.0635	0.0795	1.30	2.14
501-520	1:2	Rough	1.69	"	"	"	"	"	0.0492	0.0616	1.40	1.50
521-540	1:2	Rough	1.69	1.00	4.98	0.260	0.0522	0.339	0.0427	0.0427	0.63	1.14
541-556	1:2	Smooth	1.69	"	"	"	"	"	0.0542	0.0542	0.80	1.45
557-573	1:2	Smooth	1.69	0.75	2.87	0.16	0.0558	0.589	0.0165	0.0128	0.50	1.18
574-584	1:2	Rough	1.69	"	"	"	"	"	0.0142	0.0110	0.43	0.92

* The values obtained from Figures 3 and 4 as maximums from experimental curves.

Experimental Results

A consideration of the various terms shows that the following dimensionless groupings can be used to represent the relationship between the water passing over the structure and the other variables:

$$TQ/H^2 = f(h/H;\ d/L;\ H/L;\ \theta)$$

where

- Q = water passing over the structure by wave action in cfs per foot of structure ($ft^2/sec.$)
- T = wave period in seconds
- L = wave length in feet
- H = wave height in feet
- h = elevation of the structure in feet above (or below) the still-water level.
- θ = slope of the structure

To demonstrate the relative effect of the variables, the test data are summarized in Figs. 3 and 4. Figs. 3 a-b and 4 a-b show for slope 1:2 and 1:3 respectively, the rate of overtopping as a function of crest height for two values of relative depth and a constant wave steepness. Figs. 3 c-e and 4 c-e show the rate of overtopping as a function of crest height for three wave steepnesses and a constant relative depth.

The effect of the relative crest elevation of the structure above or below the still-water level is demonstrated in Figs. 3 and 4. Smooth curves have been drawn for all the various test conditions. In general the shape of all the curves is very similar. The maximum overtopping seems to occur when the crest of the structure is slightly above the still-water level with an average value of h/H approximately 0.2. From this point on, the amount of overtopping water decreases as the crest of the structure is placed further below the still-water level. These results apply naturally only for the conditions where the still-water level is at the same elevations on both sides of the structures, which is the case for most harbor protection structures, reefs, etc. The conditions where there is no water on the lee side of the structure would be the case of flow over a weir combined with surface wave action. This condition has been treated previously by Gibson (1930).

As can be seen in Figures 3 and 4, the roughness of the face of a structure has a considerable effect on the amount of overtopping water. At the maximum rate of overtopping, the rough surface results in an approximately 20 percent less amount than for the smooth surface. For higher values of h/H this percentage is even higher. In general, the empirical curves indicate that the rate of overtopping on a structure with relatively rough sides may be 20 percent lower than on that with smooth sides. In the case where the edge of the structure is below the still-water elevation, the roughness does not influence the results appreciably. It seems even that in this region the rough surface results in higher overtopping for some cases. This seems to be logical, for a rough surface prevents the backflow of water more than a smooth surface.

Relative depth of water: d/L is also a factor which would influence the rate of flow over a reef or structure by wave action. The steepness of the waves approaching shallow water depends on the relative depth of the water, and so does the critical steepness given by the formula:

$$(H/L)_{critical} = 0.140 \tanh 2\pi d/L$$

A wave which is near the critical steepness for breaking can be easily "tripped" and made to break by any outside disturbance. The location of the breaking point was found to be an important factor in the overflow by wave overtopping. When the wave broke just over the edge of the structure, the discharge caused by wave overtopping was relatively high. On the other hand, when breaking occurred before the wave reached the structure, considerable energy was lost and the discharge by overtopping was relatively small.

Wave steepness: To compare the results obtained under various wave steepnesses, but with a constant d/L, the curves from Figures 3 c-e and 4 c-e were replotted in Figure 5. A comparison of the various curves shows that the maximum rate of overtopping generally was greatest for the waves of small steepness. This is more evident in Figure 6 where the maximum values of QT/H^2 as obtained from the curves in Figure 5 are plotted as a function of wave steepness, H/L. These curves indicate an increasing value for the maximum value of QT/H^2 as the wave steepness decreases. There appears to be a certain wave steepness which results in a maximum value of $(QT/H^2)_{max.}$, however, considerably more data are necessary to draw more definite conclusions. Figure 6 indicates also a higher discharge of water for a structure with a side slope of 1:3 than for one with a slope of 1:2.

Wave run-up: To supplement the previous work on wave run-up by Granthem (1953), data have been obtained from Figures 3 and 4. Wave run-up in this instance was equal to the elevation of the crest of the structure when the latter was just high enough to prevent overtopping. Thus, for the data shown in Figures 3 and 4 the wave run-up was computed from the relative crest elevation, h/H, at the point where the curve showed a zero discharge by overtopping. The computed values are plotted in Figure 7 as a function of wave steepness, H/L, for slopes 1:2 and 1:3 and for smooth and rough surfaces of the structure. Figure 7 indicates that there is a critical wave steepness which results in the highest run-up of the waves. The wave steepness determines the breaking point, and when breaking occurs just at the edge of the structure the run-up is a maximum. When the breaking point moves seaward, the run-up decreases. When the waves do not break on the structure, the run-up decreases again with a decreasing wave steepness, and it is expected that it would reach the half wave height for very long waves (such as the tide). This is confirmed also by the fact that the curves in Figure 7 indicate a trend toward the value h/H = 0.5 for zero steepness. Figure 7 indicates also that there is a different critical steepness for different structures. For the slope 1:2, this critical H/L value is higher than for a 1:3 slope for the given wave condition.

As shown in Figure 7, the roughness of the structure also has an important bearing on the wave run-up. Thus run-up is about 30 percent higher for a smooth surface as compared with a rough surface.

Conclusions

Within the limits of the experiments on uniformly sloping impermeable structures, it is concluded that:

1. For a given relative height of a structure and a given relative depth, the overtopping of water caused by wave action is largest for the structure with a smooth face.

2. The maximum rate of overtopping occurs when the structure is approximately 1/5 of the wave height above the still-water level, provided that the still-water level is at the same elevation on both sides of the structure. For a given relative depth and surface roughness, the maximum rate of overtopping decreases as the wave steepness increases.

Acknowledgments

This study was part of an extensive investigation on waves and related phenomena sponsored by the Office of Naval Research.

The author wishes to express his appreciation of the suggestions, supervision and assistance of Professor J.W. Johnson in carrying out the experiments and presenting the results.

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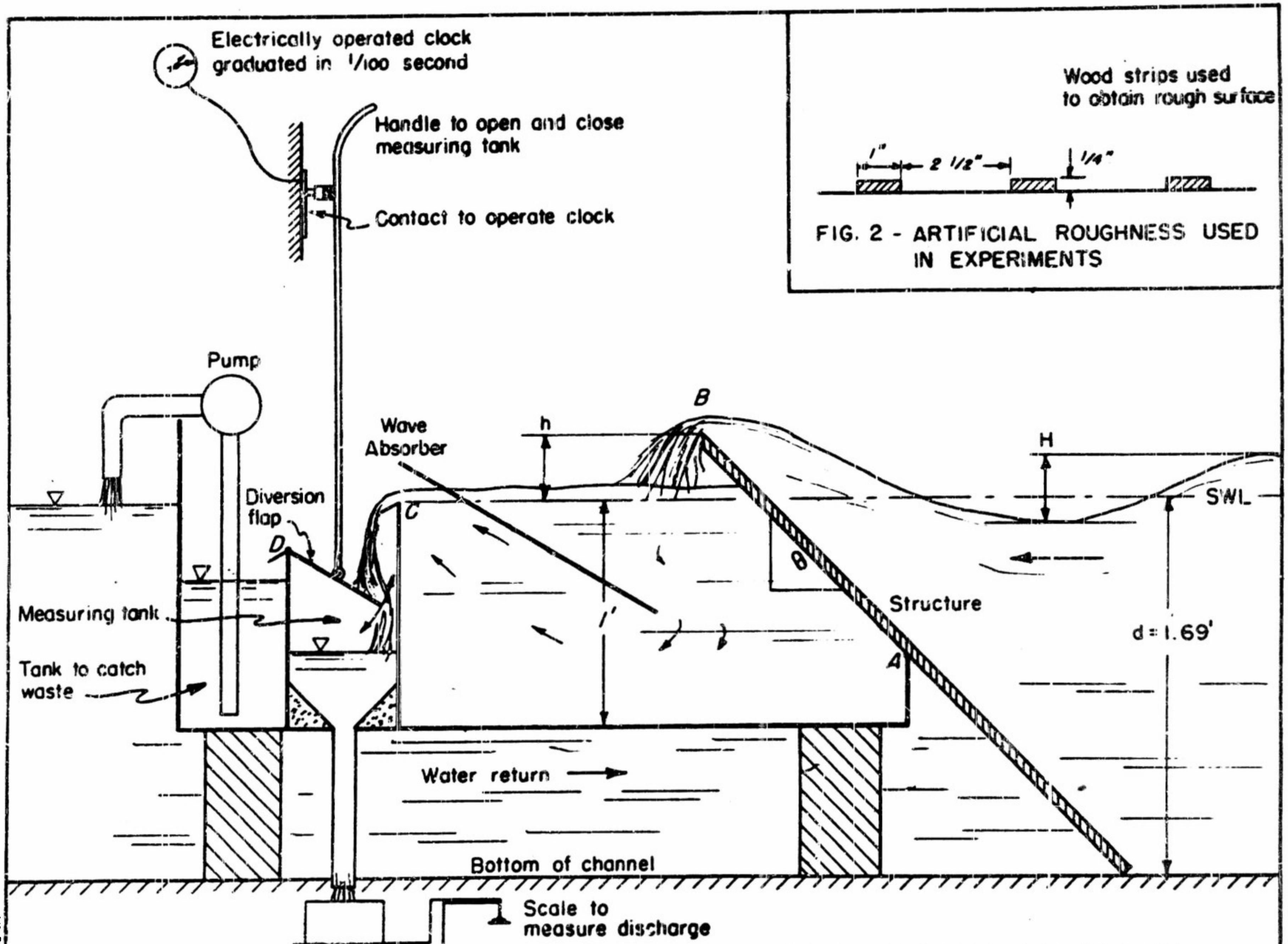
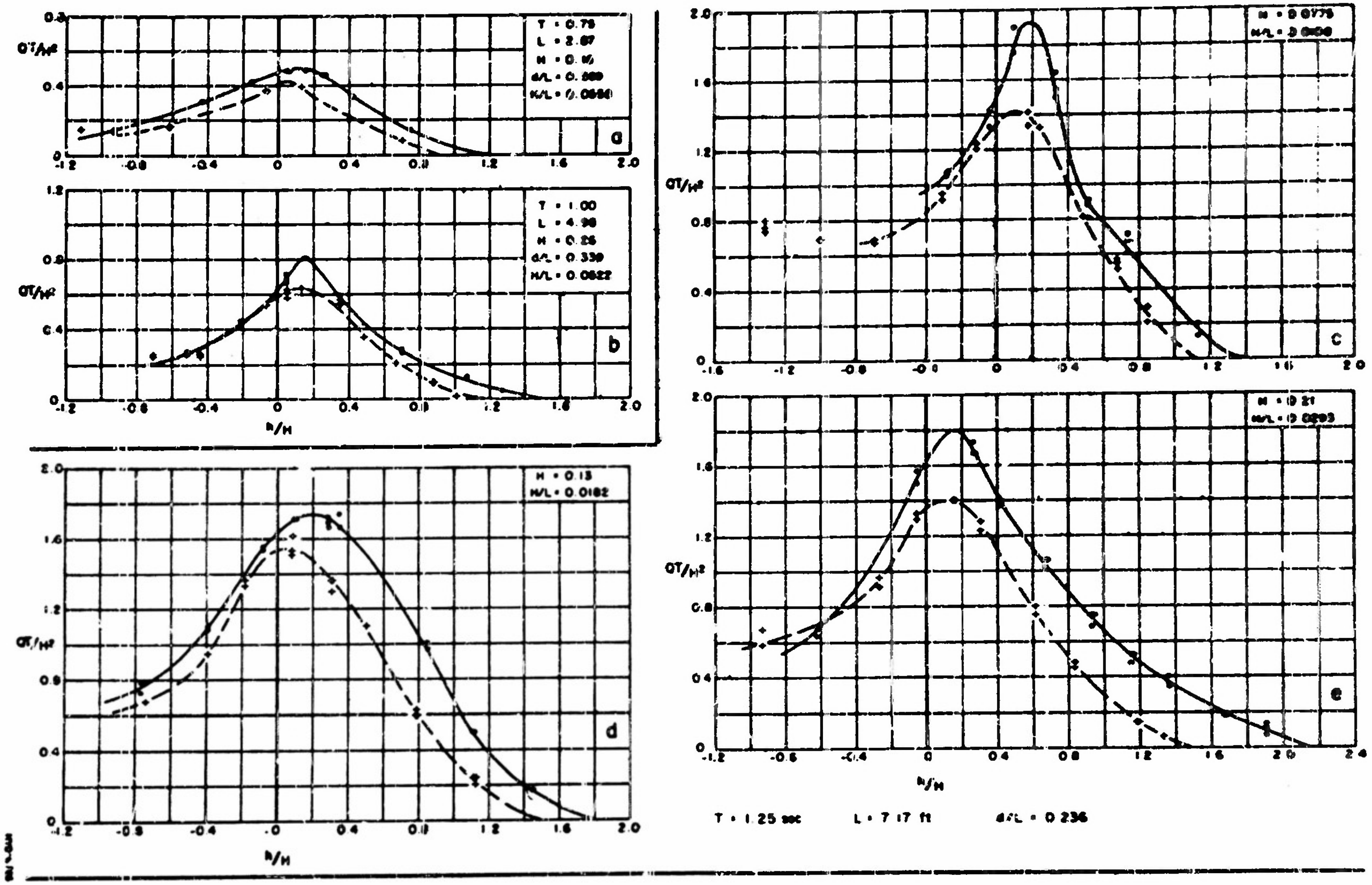


FIG. 2 - ARTIFICIAL ROUGHNESS USED IN EXPERIMENTS

FIG. 1 - SKETCH OF SET-UP



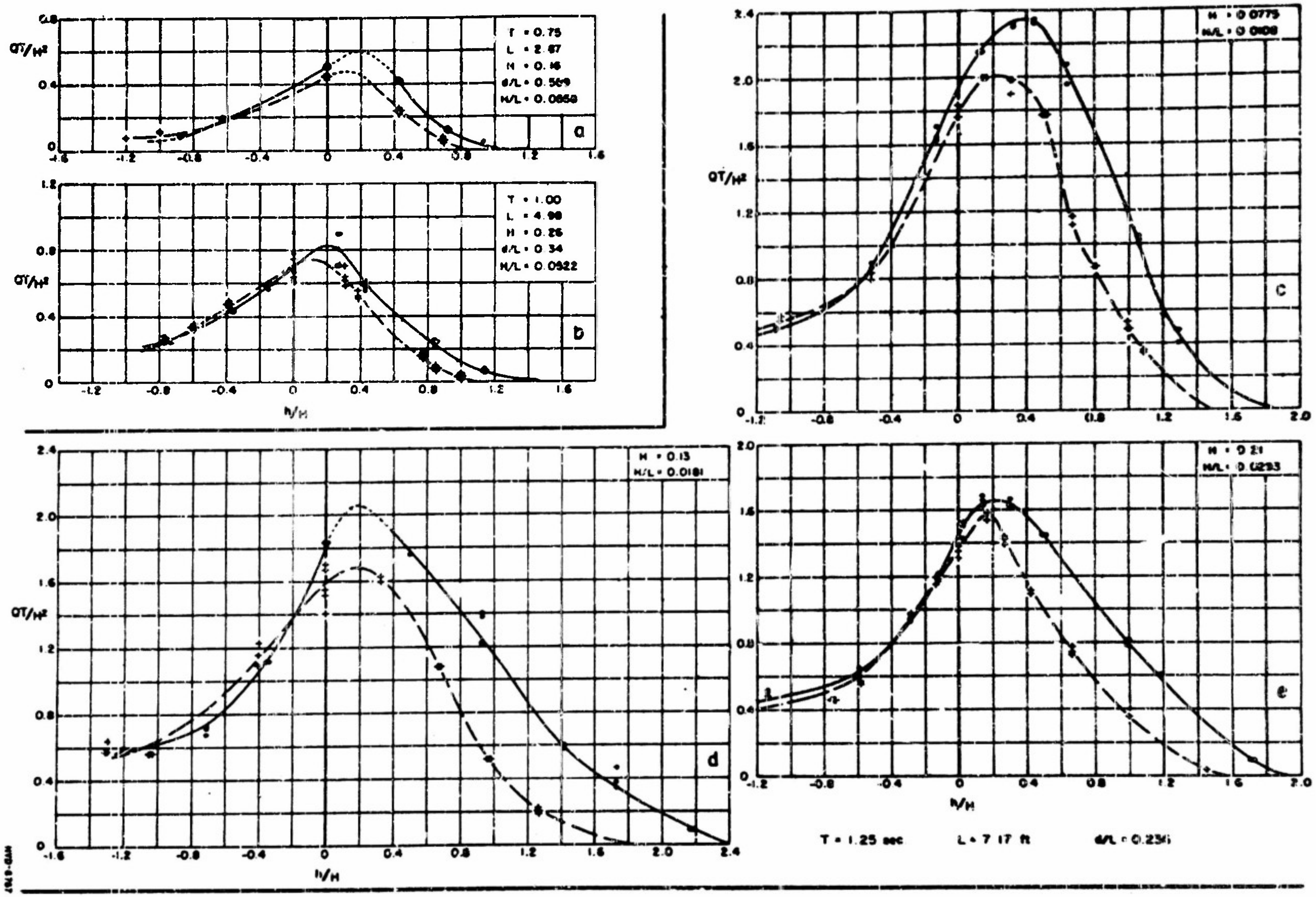
WAVE OVERTOPPING ON MARINE STRUCTURE AS A FUNCTION OF RELATIVE HEIGHT OF STRUCTURE

Side slope, θ = 1:2

Depth of water, d = 1.69 feet

Smooth surface — Rough surface

FIGURE 3

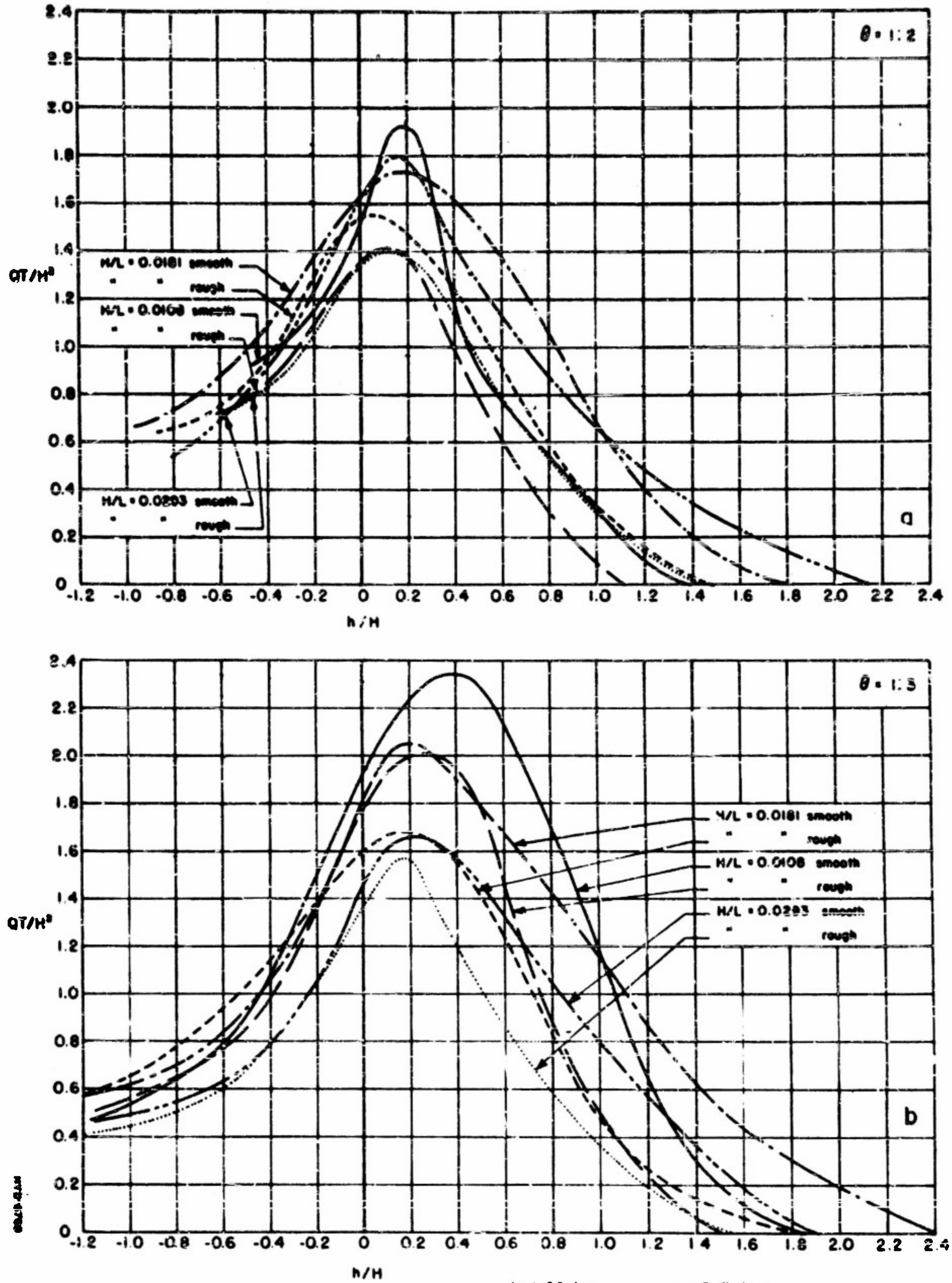


WAVE OVERTOPPING ON A STRUCTURE WITH UNIFORMLY SLOPING IMPERMEABLE SIDES

Side slope, θ = 1:3 Depth of water, d = 1.69 feet

—·— Smooth surface —+— Rough surface

FIGURE 4



d = 1.69 feet   L = 7.17 feet
T = 1.25 seconds   d/L = 0.236

WAVE OVERTOPPING AS A FUNCTION OF HEIGHT OF STRUCTURE ABOVE STILL WATER FOR CONSTANT d/L RATIO AND VARIABLE WAVE STEEPNESS

FIGURE 5

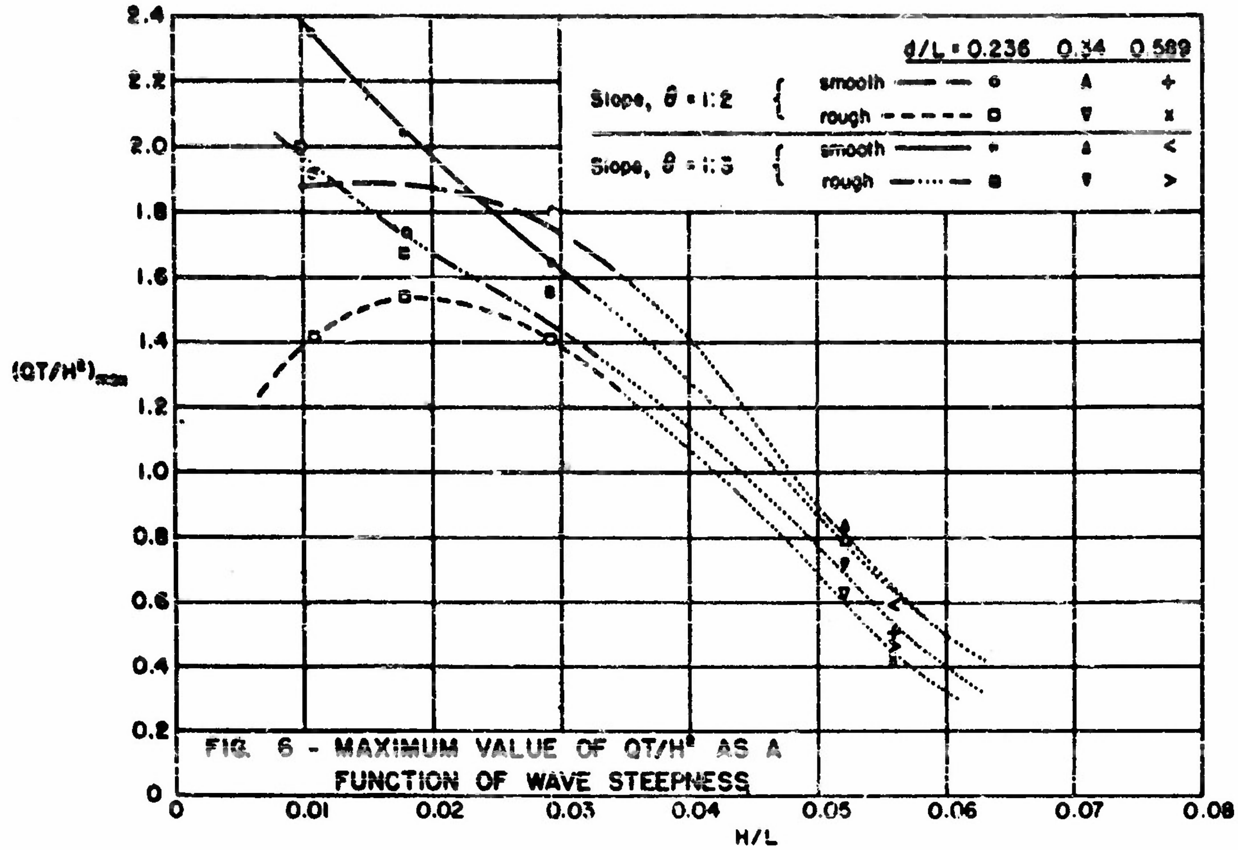


FIG. 6 - MAXIMUM VALUE OF QT/H² AS A FUNCTION OF WAVE STEEPNESS

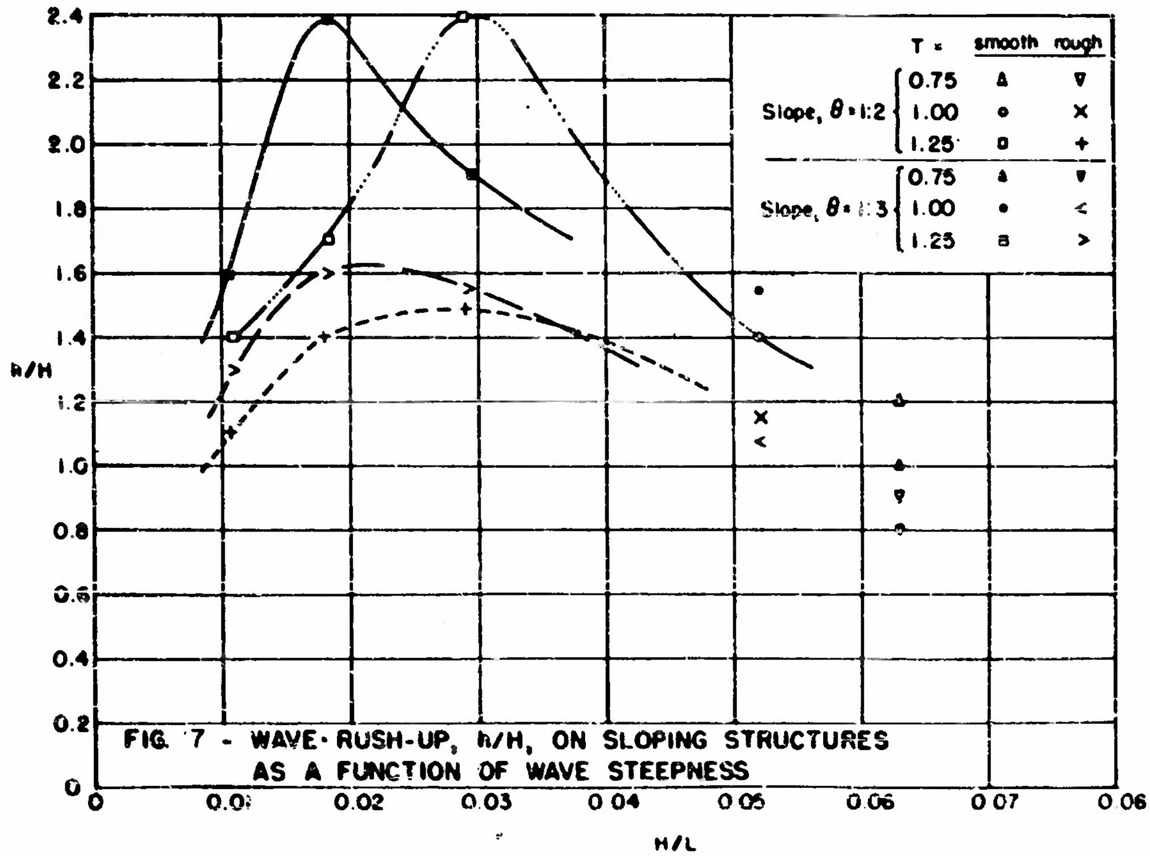


FIG. 7 - WAVE RUSH-UP, h/H, ON SLOPING STRUCTURES AS A FUNCTION OF WAVE STEEPNESS

FIGURES 6 & 7

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